STA-5 2-D Hydraulic Modeling
(Linked Cells Model)
Task 4.3 Final Report

Science and Engineering Support Service (SESS)
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South Florida Water Management District

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Table of Contents

Table of Figures

STA-5 2-D Hydraulic Modeling (Linked Cells Model) Task 4.3 Final Report

1. Introduction

Stormwater Treatment Area 5 (STA-5) is a primary component of the Everglades Construction Project mandated by the 1994 Everglades Forever Act (section 373.4592, Florida Statutes). It is located immediately north of U.S. Sugar Corporation’s Southern Division Ranch, Unit 2, and extends from the L-2 borrow canal on the west, to the Rotenberger Wildlife Management Area on the east. STA-5 provides a total effective treatment area of 4,110 acres to treat stormwater runoff originating within the C-139 Basin. The location of STA-5 is shown in Figure 1.

This document describes the development of a two-dimensional linked cells hydraulic model for STA-5. Previous 2-D hydraulic models of STA-5 (Burns and McDonnell, 2004) were single cell models based on the 1997 design data including the topographic data available at that time. Since then, a new topographic survey has been completed, and structural and vegetative enhancements have been initiated in the STA. Under this current modeling effort, a STA-5 linked cells model was built from scratch using revised topographic, vegetation and structural data.

Model calibration and validation with historic stage and flow data were not performed at the request of the District. The reason is that STA-5 is currently under significant enhancements and model parameters calibrated to current configuration may not necessarily represent the enhanced STA-5 hydraulics.

The new linked cells model is used to simulate steady flow scenarios for STA-5 for Low, Design and High Flow Conditions under Enhanced configurations. The majority of present tasks are spelled out under Task 4 of the contract scope of work, precisely under Subtask 4.1: STA-5 Linked Cells Model.

This final report (Subtask 4.3) summarizes major results obtained in the modeling work for Subtask 4.1 and comments from District staff have been incorporated based on the draft report (Subtask 4.2).

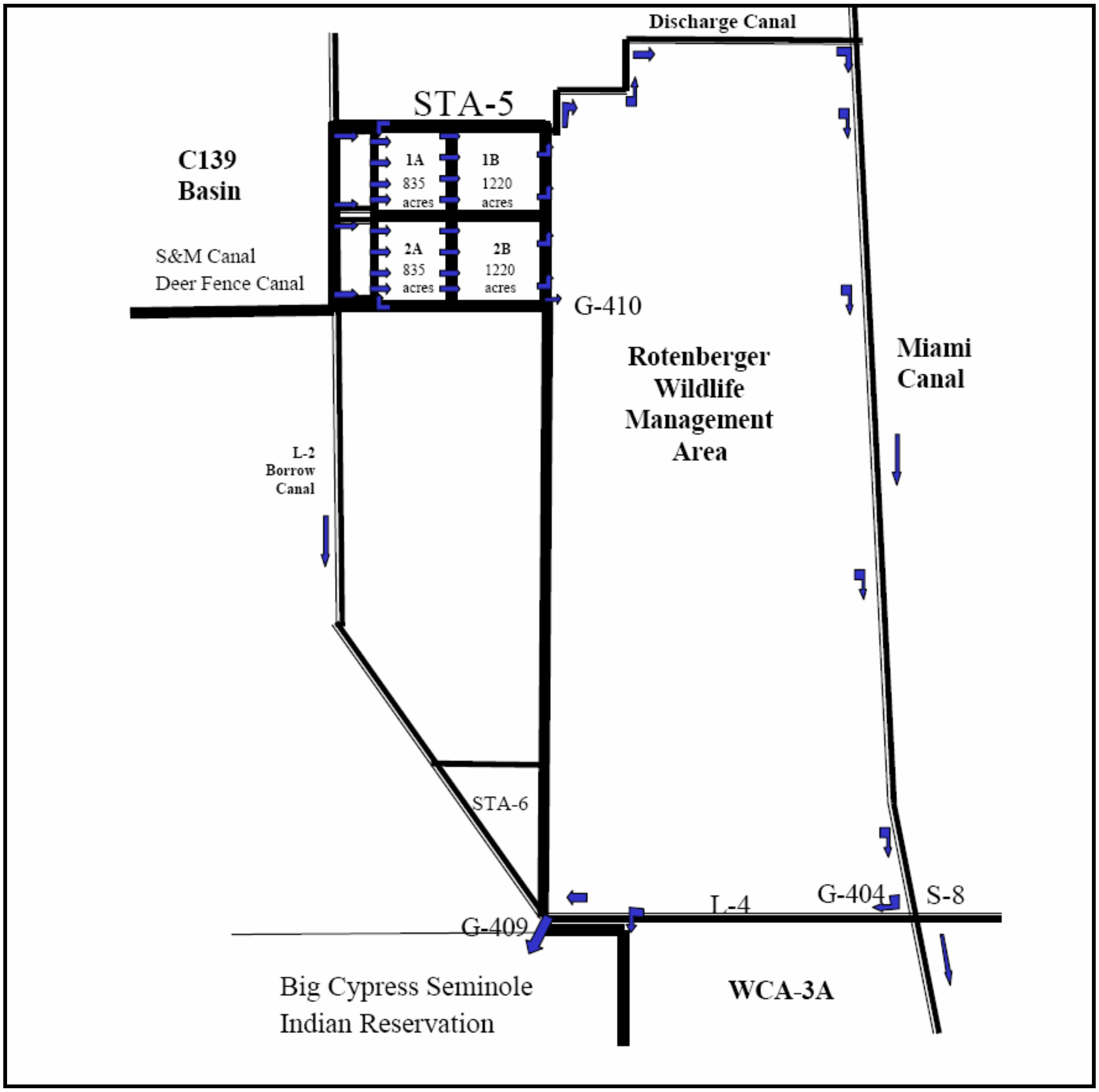

Figure 1: Schematic of STA-5 and Surrounding Area (not to scale)

2. Model Setup

2.1 Modeling Tool

The FESWMS/FLO2DH computer program was selected by the District as the modeling tool for the current hydrodynamic modeling of STAs. The Flo2DH model engine is part of the Federal Highway Administration's Finite Element Surface-water Modeling System (FESWMS). It is a public domain model code but the Graphic User Interface (GUI) through the Surface Water Modeling System (SMS) is commercial software. FLO2DH simulates two-dimensional depth averaged hydrodynamic flows of surface water bodies using the finite element method. Additional information about the theoretical background of the model code, its numerical method, input and output data requirement can be found in the User’s Manual for FLO2DH 3.0 (Froehlich, 2002). The version of FESWMS/FLO2DH used in this modeling work is 3.2.0.

2.2 Conceptual model

STA-5 is currently undergoing major modifications and enhancements. Work on these modifications and enhancements began in early 2005 and will continue into late 2006. The current modeling effort to develop a linked cells model for STA-5 was based on the following information.

The major modifications and enhancements are based on the revised Long Term Plan for Achieving Water Quality Goals (SFWMD, 2004):

- Removal of flow obstructions in Cells 1B and 2B observed directly upstream of the G-344 structures.
- Conversion of Cell 2B from emergent cattail vegetation to submerged aquatic vegetation (SAV).
- Modification of G-343 structures: the addition of operable gates and the upstream weir controls removed.

The above modifications were incorporated into the new linked cells model as follows:

- Removal of flow obstructions in Cells 1B and 2B: land surface elevations immediately upstream of G-344A-D were assumed to be leveled down to the elevation that is close to the surrounding grade.
- Cell 2B vegetation was set as SAV.
- G-343A-H structures: simulated as culverts instead of weirs.

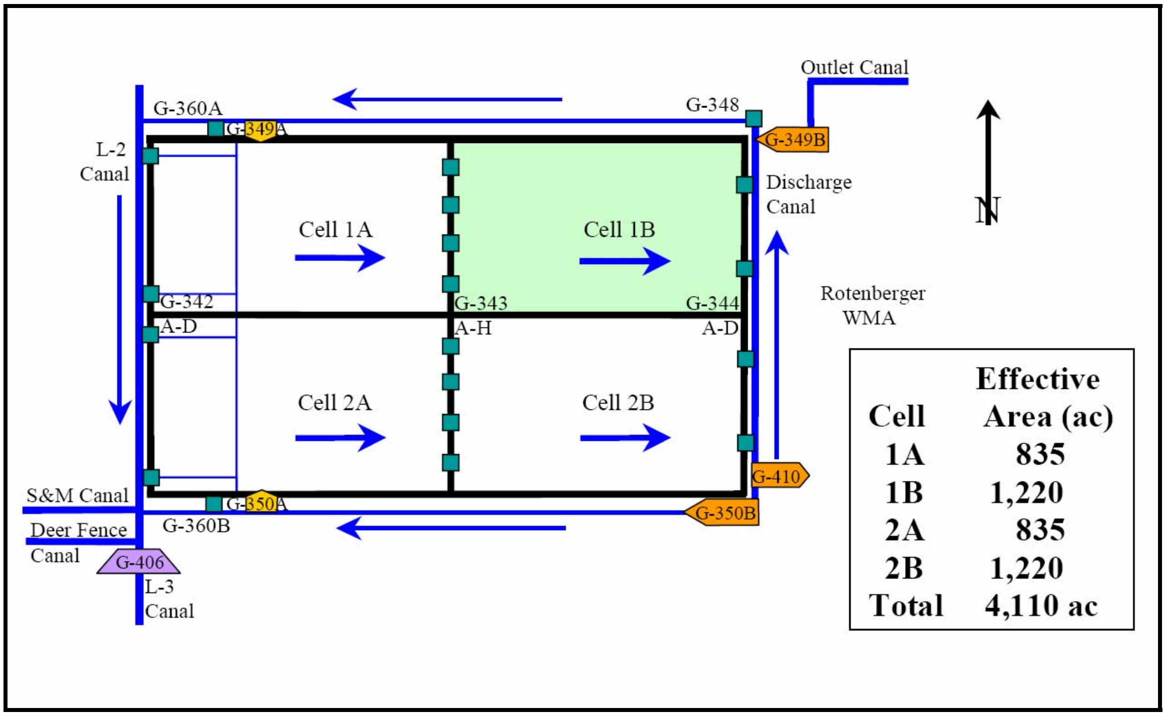

Figure 2: STA-5 Layout (not to scale)

A schematic of STA-5 is shown in Figure 2. Stormwater runoff from the C-139 Basin enters the L-2 borrow canal and is conveyed from the G-342A-D structures into STA-5. The diversion structure G-406 is normally closed to direct runoff to STA-5 for treatment. When opened, it routes runoff to the south.

STA-5 is divided into two separate flow-ways by interior levees. The northern flow-way consists of Cells 1A and 1B. Water flow into Cell 1A is controlled by the gated inflow culverts G-342A and G-342B. Stage and flow in Cell 1A can be controlled by interior gated culverts G-343A-D. Out flow structures for this flow way is the gated culverts G-344A and G-344B. Similarly, inflow into Cell 2A is controlled by gated inflow culverts G-342C and G-342D. Water flow into Cell 2B via interior gated culverts G-343E-H and eventually treated water is discharged into the Discharge Canal through G-344C and G-344D.

STA-5 Discharge Canal is connected to the Miami Canal in the downstream. Pump station G-410 can also transfer water from STA-5 discharge canal to the Rotenberger Wildlife Management Area.

Pump stations G-349B and G-350B can be used to provide supplementary water supply to avoid STA-5 dry-out under drought condition. Structures G-360A and G-360B are located in the western sections of the north and south seepage canals to control drawdown in the seepage canals. These pump stations and structures are not simulated in the 2-D hydraulic model.

In the linked cells model, sixteen culverts are all explicitly represented by FESWMS-FLO2DH’s culvert option: G-342A-D, G-343A-H and G-344A-D. The diversion structure G-406 is acting as a boundary for the L-2 Borrow Canal. Seepage return pumps G-349A and G-350A are simulated as point sources.

The geometry and parameters of culvert structures were obtained from the District DBHYDRO database and STA-5 Operation Plan.

All sixteen culverts are reinforced concrete box culverts (RCB). Major information about these culverts is listed in Table 1.

Table 1: Information for STA-5 Culverts

Culvert Name	G-342A-D	G-343A-H	G-344A-D
unit	4	8	4
Type	RCB	RCB	RCB
Entrance loss coefficient	0.5	0.5	0.5
Manning’s n	0.012	0.024	0.012
Invert	7.25 ft	5.49 ft	0.0 ft
Flow line length	68.0 ft	60.0 ft	53.0 ft
Size	10 ft x 6 ft	10 ft x 8 ft	10 ft x 10 ft

Besides structure inflows (inflow structures and seepage return pumps), the following source/sink terms are part of STA-5 water budget:

- Rainfall
- Evapotranspiration (ET)
- Levee/vertical seepage losses

In a recent STA-5 water budget study, Parrish and Huebner (Parrish and Huebner, 2004) reported that from May 1, 2000 to April 30, 2003, flow from gated culverts at G-342A-D constituted 73 percent of the total inflow to STA-5. Rainfall was 8 percent of the total inflow. Seepage return flow from G-349A and G-350A was 19 percent of the total inflow. G-344A-D outflow is 63.5 percent of the total outflow. ET and seepage losses were estimated to be 10.7 percent and 25.8 percent, respectively. These percentages are expected to be varied from year to year.

For flow scenario simulations, Design Flow is a rare event which has not occurred in STA-5 operation, it is expected to be of short duration, so rainfall and ET were not considered (in the wet season, they are likely cancelled out) and seepage along levee was assumed as 20% of structure inflow. High Flow (Standard Project Storm) is an extreme event lasting for a few days, only the design rainfall is considered, ET and seepage are neglected. As for Low Flow, rainfall is negligible; ET was assumed to be 0.24 inch/day and seepage losses were assumed to be 60 cfs, 30% of the G-342A-D inflow.

The L-2 Borrow Canal and the Discharge Canal are included and they are connected to the treatment cells via inflow and outflow structures G-342A-D and G-344A-D, respectively.

2.3 STA-5 topography and finite element mesh

The latest STA-5 topographic survey data (Wantman Group, 2005) was provided by the District for this modeling study. The newly surveyed land surface elevations inside the treatment areas range from 10.0 ft NGVD to 16.0 ft NGVD (marsh areas). The average ground elevation in Cell 1A, excluding the non-effective treatment area on the west, is 13.38 ft NGVD and the average ground elevation in Cell 1B is 11.85 ft NGVD; The average ground elevation in Cell 2A, excluding the non-effective treatment area on the west, is 12.83 ft NGVD, and the average ground elevation in Cell 2B is 11.99 ft NGVD.
The bottom elevations of L-2 Canal and Discharge Canal were assumed to be 0.0 ft NGVD in the model and spreader canals in Cell 1A and Cell 2A has a 5.0 ft NGVD bottom elevation. The distribution canals at G-343A-H and G-344A-D were set to be ranging from 4.0 to 6.0 ft NGVD. There are no indications or topographic data of borrow canals in the new topographic survey; although Burns & McDonnell (Burns & McDonnell, 2004) applied plugged borrow canals along the central interior levees between Cell 1B and Cell 2B. It was decided that no borrow canals would be applied in the new linked cells model.

The original surveyed topographic data have a few locations of extreme high elevations in Cell 2B and Cell 1A (Figure 3). For example, at the southwestern corner of Cell 2B, there are local areas with land surface elevations between 14.5 ft NGVD and 16.3 ft NGVD. This is about more than two feet higher than the average elevation of Cell 2B. These high elevation spots are close to G-343H. They will be always in dry-out condition even under Design Flow condition. Since frequent element drying/wetting greatly affects numerical stabilities, after numerical experiments, these high elevation spots were assigned an elevation value at 13.50 ft NGVD, this guarantees all elements will be under water in Design Flow condition. The impact on simulation results was insignificant (small water storage volume was added). The land surface elevations as interpolated in the finite element mesh is shown in Figure 4.

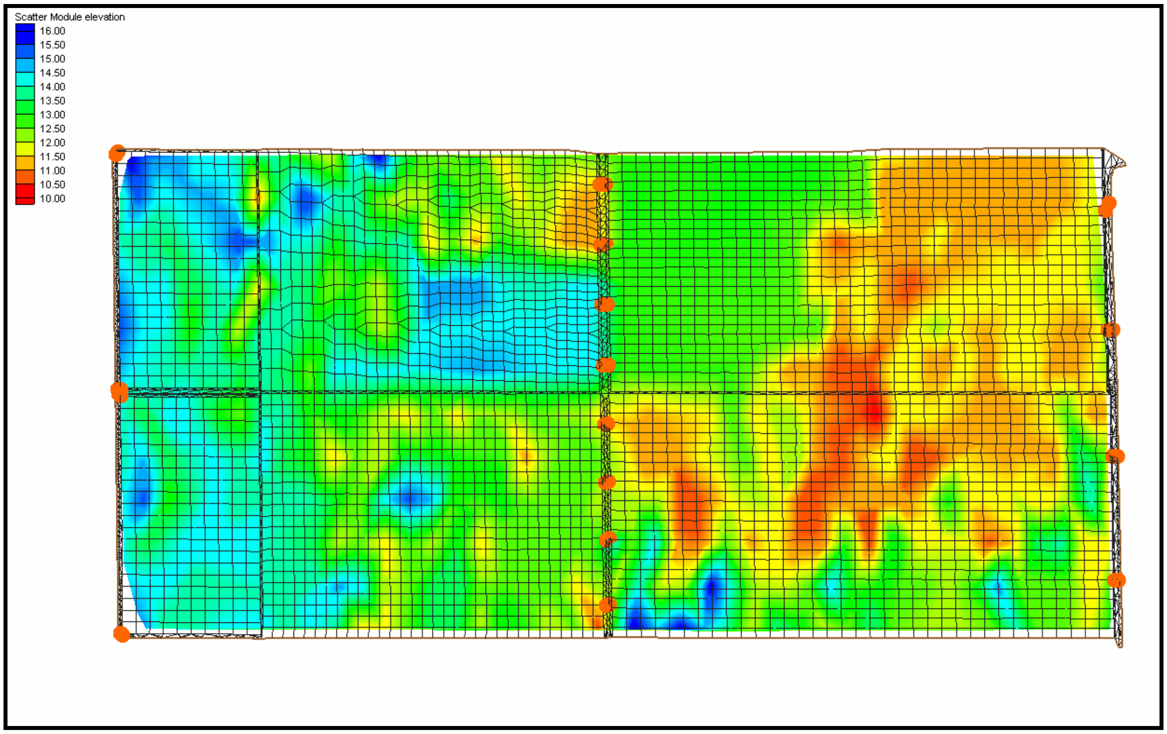

Figure 3: Land Surface Elevations in STA-5 Marsh Area (topographic survey data)

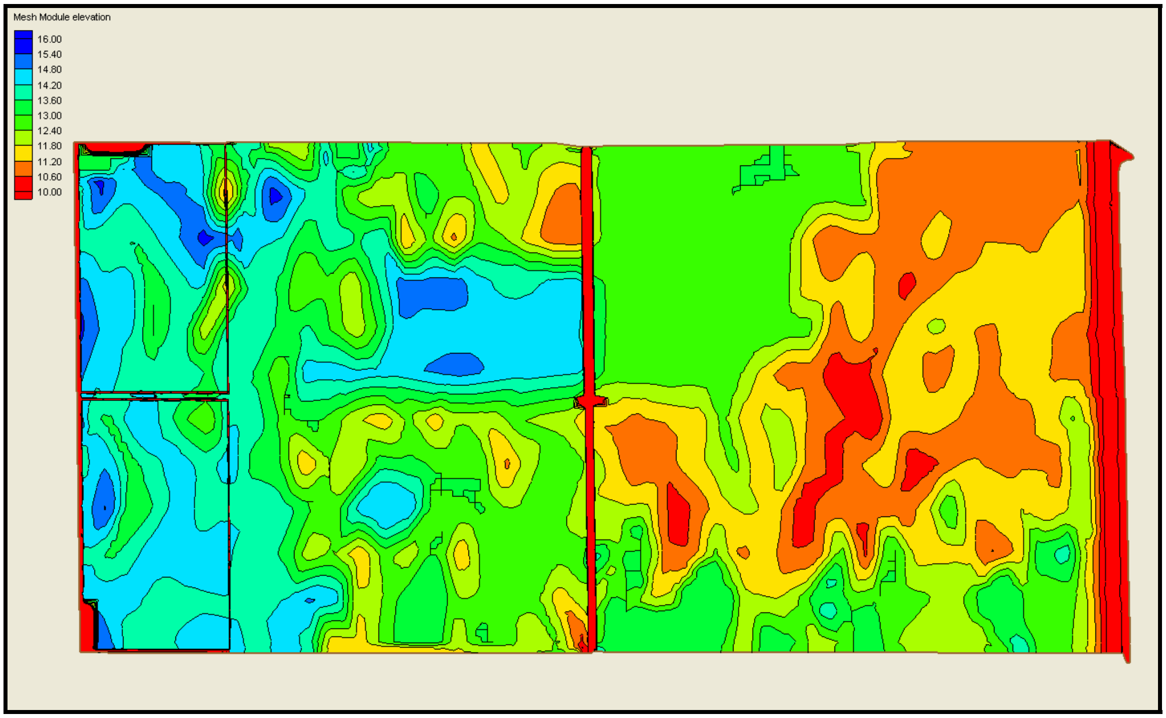

Figure 4: Land Surface Elevations used in the STA-5 linked cells model (interior levees are not included, scaled from 10 ft NGVD to 16.0 ft NGVD)

Burns & McDonnell (Burns & McDonnell, 2004) used the 1997 design topography data in a STA-5 modeling study. The difference between the 1997 topography and the more recent topographic survey by Wantman Group is significant. The difference in land surface elevations ranges from about -1.0 to 3.0 ft (Figure 5). The new topographic survey data are considered more accurate.

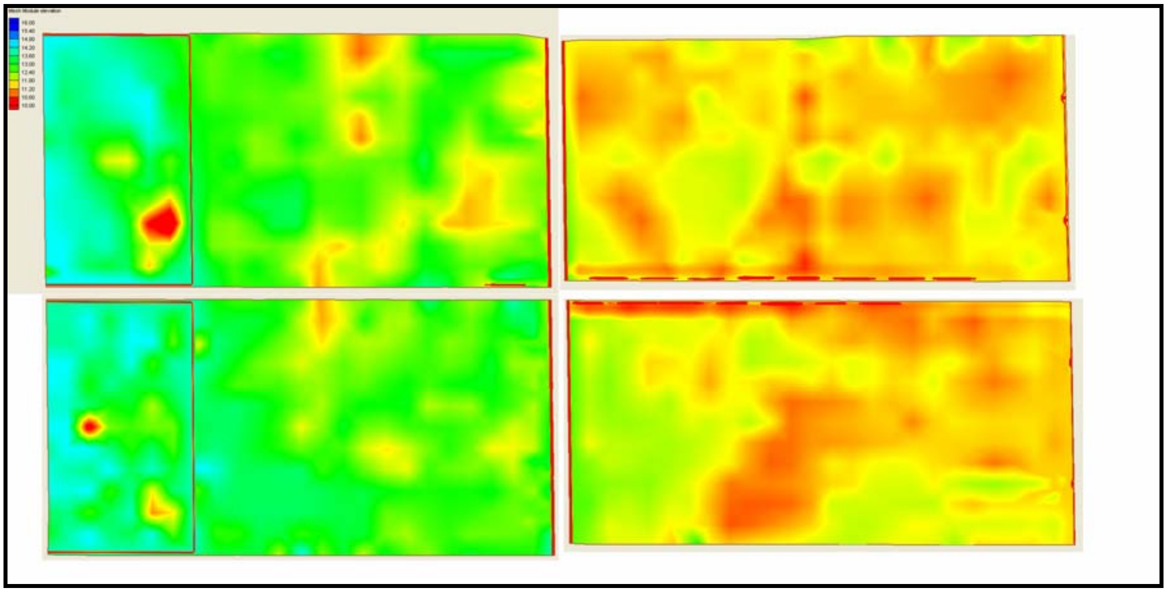

Figure 5: Land Surface Elevations Used in Previous STA-5 Modeling (Burns & McDonnell, 2004)

2.4 Material types and Manning's n value

For the current linked cells modeling effort, the spatial distribution of vegetation types were based on the 2003 vegetation map (SFWMD, 2003) with the exception that SAV is assumed to be the dominant vegetation type in Cell 2B as is proposed in the revised Long-Term Plan (SFWMD, 2004).   This is shown in Figure 6.

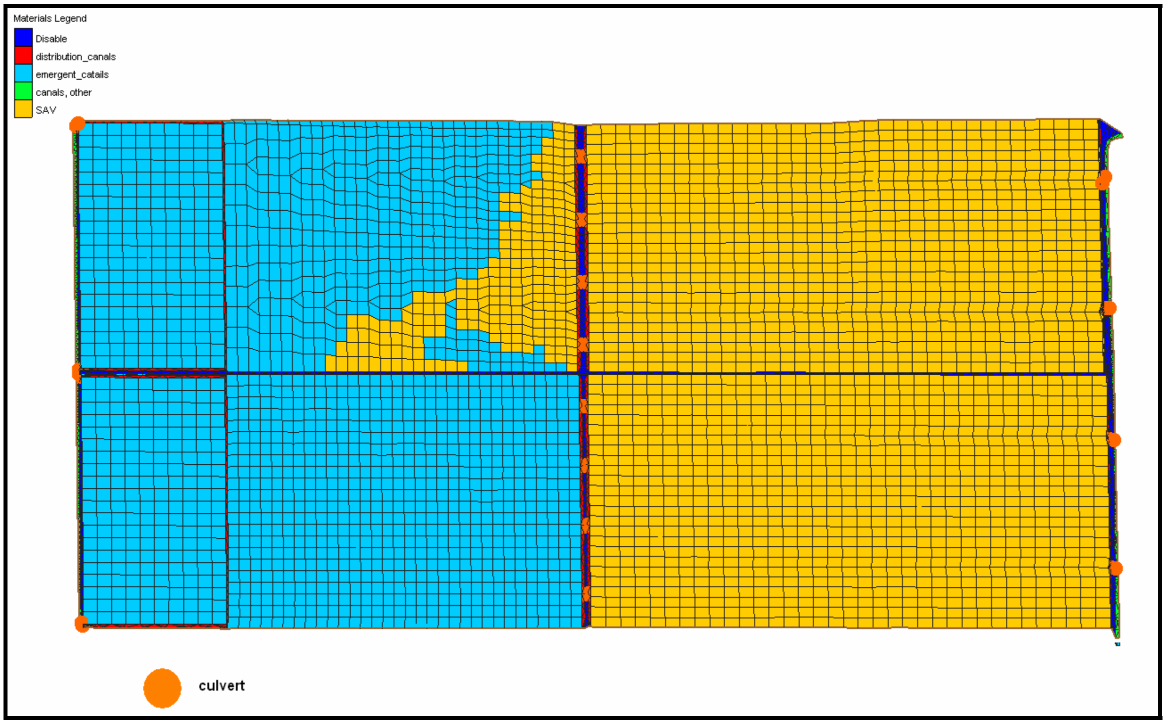

Figure 6: Material types and finite element mesh used in the STA-5 linked cells model

Due to the fact that major modifications and enhancements are underway in STA-5, including the removal of flow obstructions within the treatment cells, and because the revised topographic data is so different from earlier estimates, it was proposed that detailed model calibration and verification based on the current STA-5 configuration were not appropriate.

Since Manning's roughness coefficient (n) values are closely related to vegetation types and density, these values are spatially varied and flow-depth dependent. Currently, there are no good references on the selection of Manning's n values for vegetated covers in two-dimensional hydraulic models.

In a previous STA-5 modeling study, Burns & McDonnell (Burns & McDonnell, 2004) tried to calibrate Manning's roughness coefficient values for STA-5 based on history matching from some steady flow simulations. This work was done before the new topographic survey data was available and it is generally thought that the calibrated values are affected by the differences in topography used in the simulations.

The following Manning's roughness coefficient values were used in all simulations for the current STA-5 linked cells modeling effort (Table 2). These values were selected based on experience from previous model calibration studies for STA-2, STA-6 and STA-1W (Sutron Corp., 2004a, b, c and 2005) and other STA modeling works. The values are not much different from that of previous STA-5 modeling efforts (Burns & McDonnell, 2004).

Table 2: Manning's n Values used for STA-5

Depth (ft)	Cattail	SAV	Canals
3.0	0.5	0.3	0.035~ 0.04
1.5	Varies linearly	Varies linearly	
1.0	1.3	Varies linearly	
0.5	1.3	0.8	

It is recommended that Manning's n values will be calibrated under the new STA-5 configuration with future observed flow/stage data as part of a separate work effort.

3. Simulation of Flow Scenarios

Three flow conditions: Design Flow, Low Flow and High Flow were simulated by the STA-5 linked cells model.

Design Flow is the flow condition stated in the STA-5 operation plan (SFWMD, 2000): total runoff from the C-139 Basin is 1,750 cfs; STA-5 inflow will be 1,200 cfs and 550 cfs of runoff will be bypassed through G-406.

Low Flow is defined as 200 cfs of structure inflow from G-342A-D. This represents 100 cfs of inflow for each of the two flow ways.

High Flow is the flow condition under the Standard Project Flood (SPF) in the STA-5 operation plan. The estimated runoff from the C-139 Basin is 3,440 cfs and 21.6 inches of direct rainfall in 24 hours is assumed to fall on STA-5 treatment areas at the same time period.

3.1 Historical flow data

The total daily flow of G-342A-D were obtained and plotted in Figure 7. The maximum total daily flow (October 1999-March 2005) is 878.87 cfs. It can be seen that daily storm runoff in the dry season is usually less than 200 cfs. There are many no-flow days in the data series between storm events, so the long-term average daily flow is lower than 200 cfs.

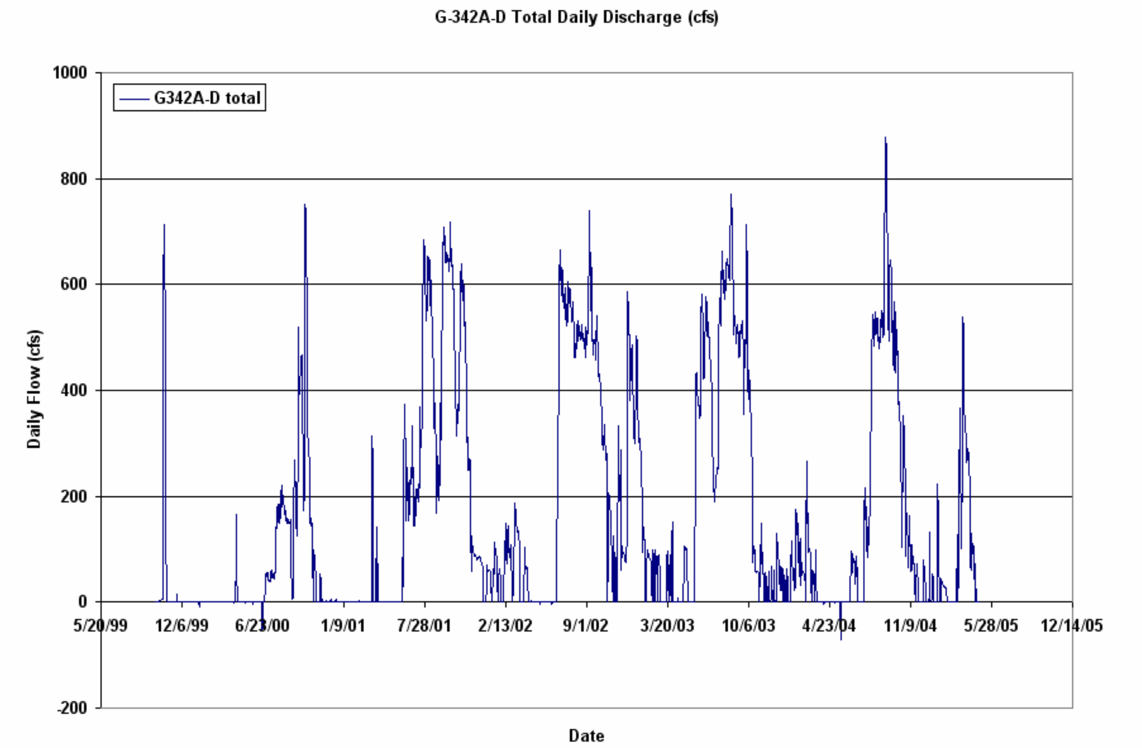

Figure 7: Total Daily Flow at G-342A-D (October 1999 to March 2005)

A preliminary frequency analysis was performed for all G-342 A-D non-zero, positive daily flow data from 1999 to 2005 (Figure 8). It can be seen that the total observed inflow at G-342A-D never exceeds the Design Flow of 1,200 cfs. And a 200 cfs (4*50, Low Flow) total inflow is at the 30%-40% of the exceeding frequency range for all non-zero, positive flows (1999 – 2005).

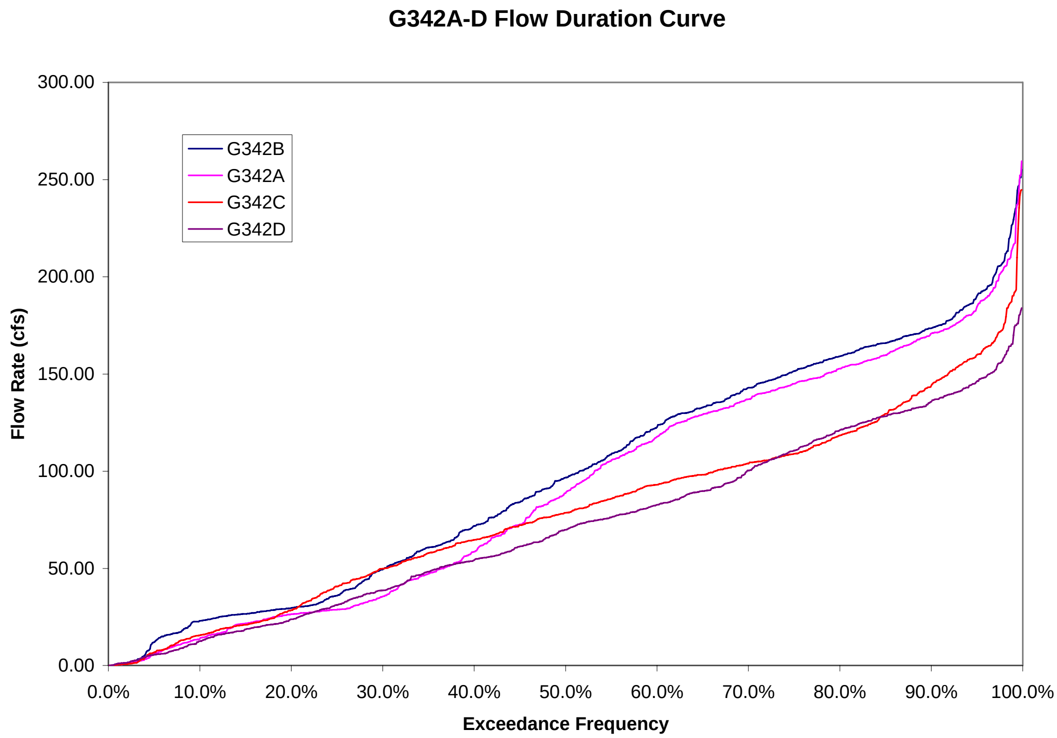

Figure 8: Frequency curves for G-342A-D daily flows (1999 – 2005)

3.2 Design Flow

- Total storm runoff into L2 Borrow Canal is 1,750 cfs.
- G-406 diverts 550 cfs. So the total STA-5 inflow is 1,200 cfs.
- And the seepage return flow is 50 cfs (2 x 25).
- Levee seepage losses were considered (20% of G-342 inflow): -2 x 120 cfs, as specified flux along the southern and northern levees.
- Downstream boundary: G-344-A TW in the Discharge Canal = 14.13 ft NGVD.

The boundary conditions and source/sink assignment for Design Flow simulation was illustrated in Figure 9.

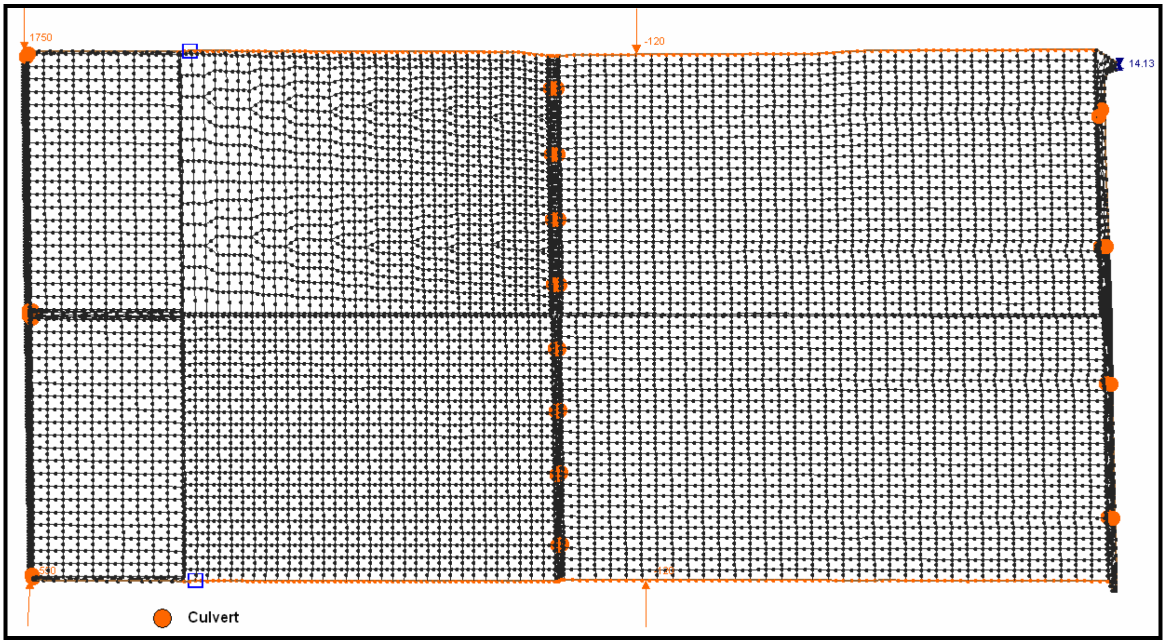

Figure 9: Simulation Setup for Design Flow

Transient simulations were made with constant boundary conditions to approach steady state. Water mass balance, stage and flow were checked for final steady state flow condition.

The final simulation results demonstrated good mass balance and the flow distribution at G-342A-D was obtained:

G-342A: 375 cfs; G-342B: 269 cfs; G-342C: 248 cfs and G-342D: 308 cfs.

In the following paragraphs, flow patterns under Design Flow condition will be discussed.

In Figure 10, transects A-A' and B-B' across Cell 1 and Cell 2 will be used to plot and compare simulation results.

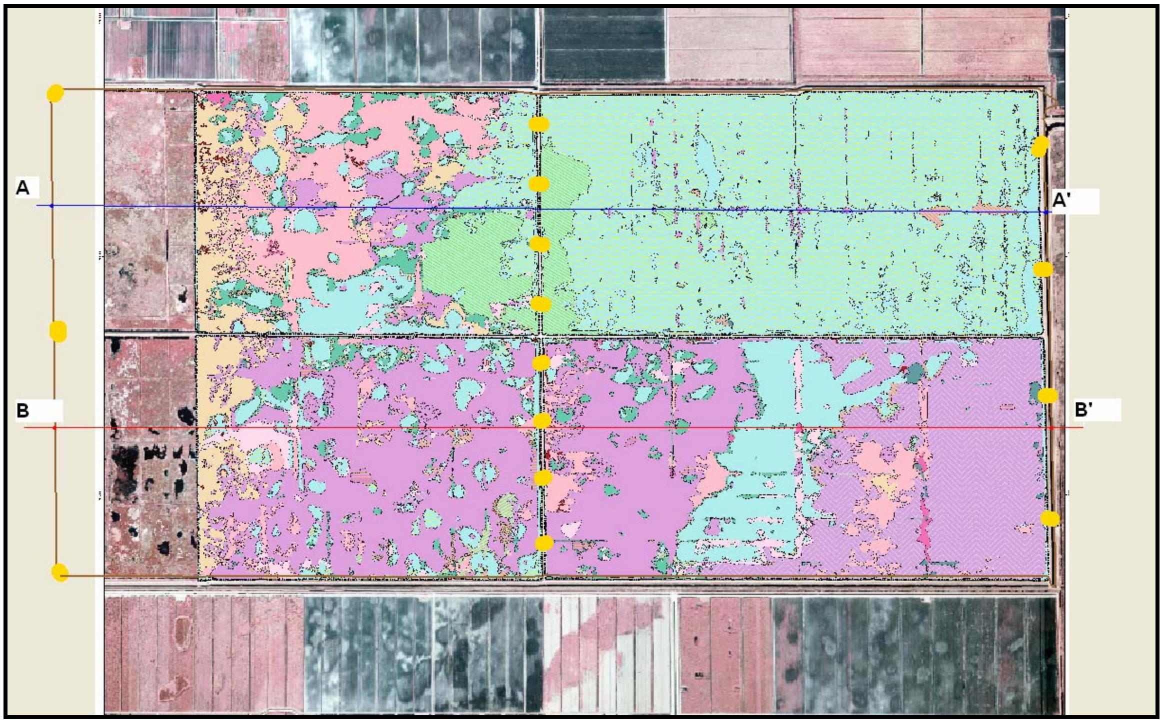

Figure 10: Location of Transects for presenting simulation results

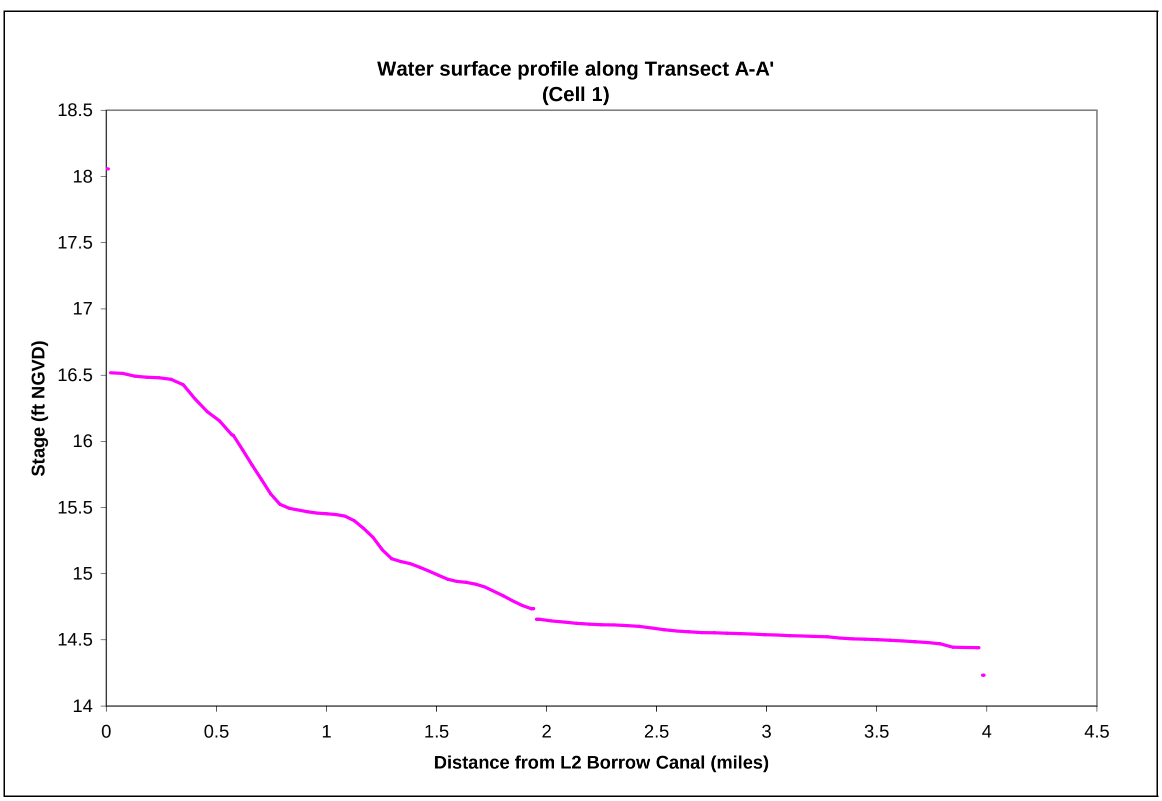

Figure 11: Water surface profile along Cell 1 (west to east, A-A').

The west portion of STA-5 (west of the spreader canals) has higher land surface elevations (ranging from 13.5 ft NGVD to 16.0 ft NGVD). This area is dry most of the time during normal operation. It is observed that under Design Flow condition, water surface elevation is 16-17 ft NGVD in this area, so there is flow in this region. Maximum historical STA-5 inflow (878.87 cfs) has not been reached the Design Flow (Figures 11 and 12).

Velocity magnitude value is under 0.1 ft/s in marsh area (Figure 13) and water depth in marsh area ranges from 0.32 ft to 4.5 ft (Figure 14). From this simulation, there is no obvious flow short-circuiting flow-ways in STA-5.

In Figure 15, the water surface profile under Design Flow condition was plotted with the 3-D surveyed topography in the marsh area. As discussed before, a few high elevation spots are in dry-out condition under Design Flow.

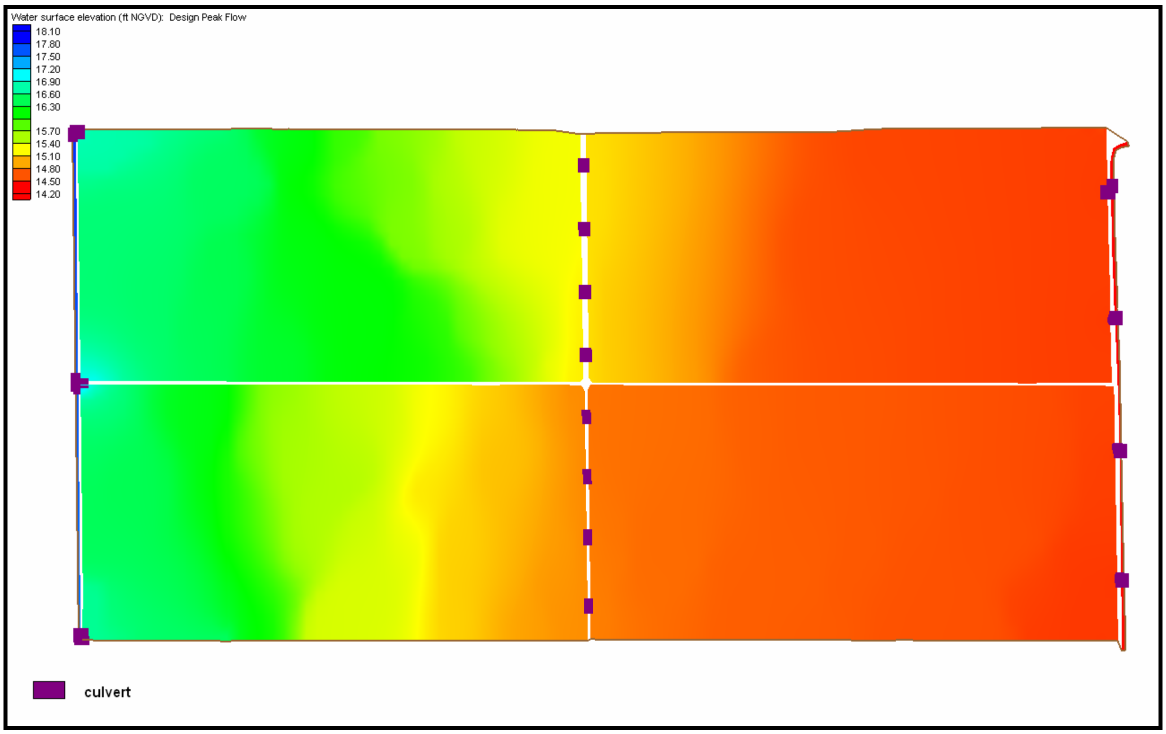

Figure 12: Water Surface Elevation under Design Flow

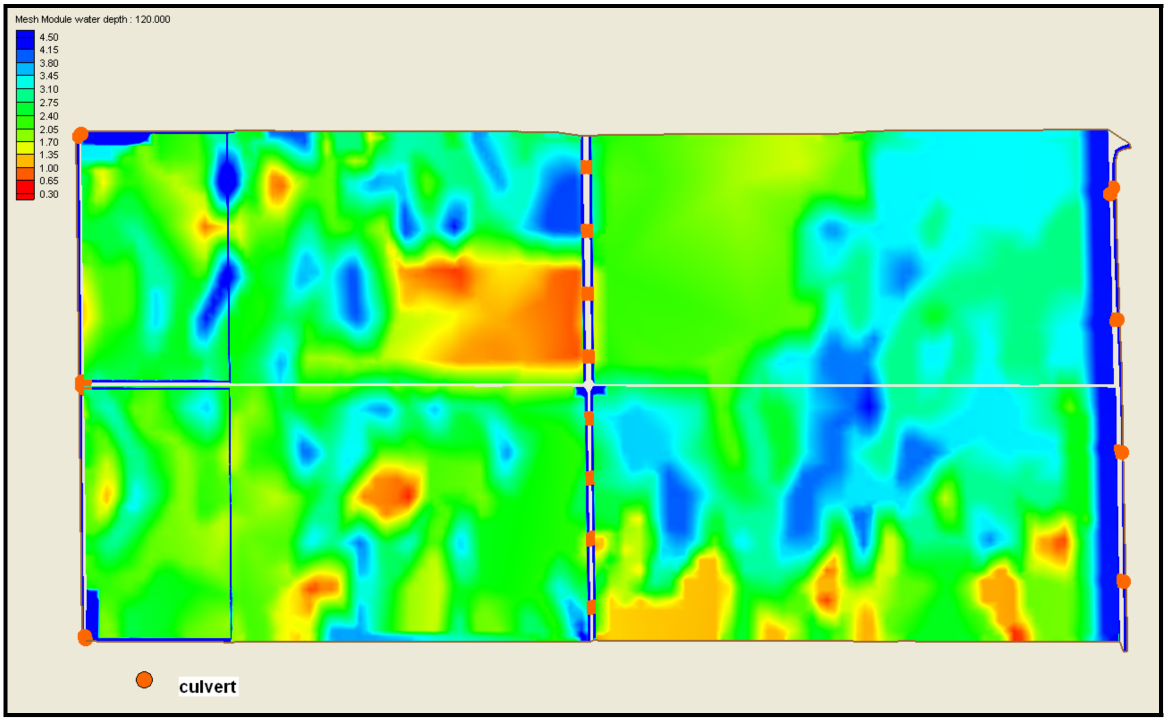

Figure 13: Water depth distribution under Design Flow

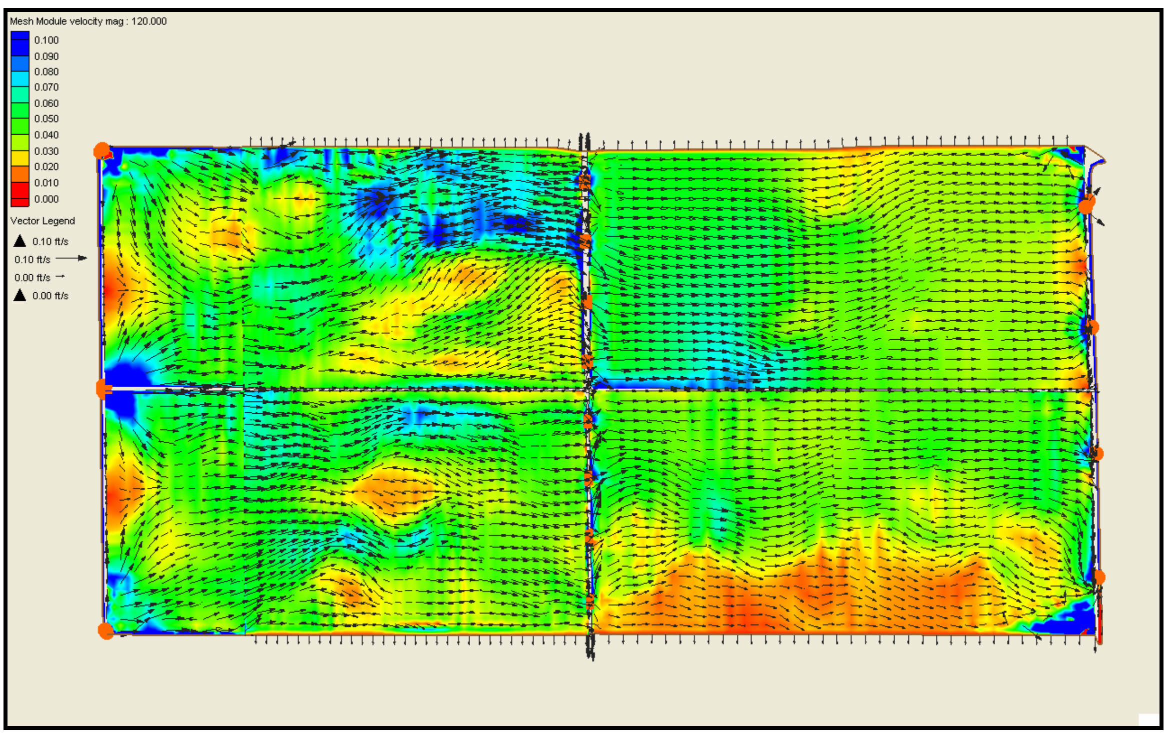

Figure 14: Velocity Magnitude distribution under Design Flow.

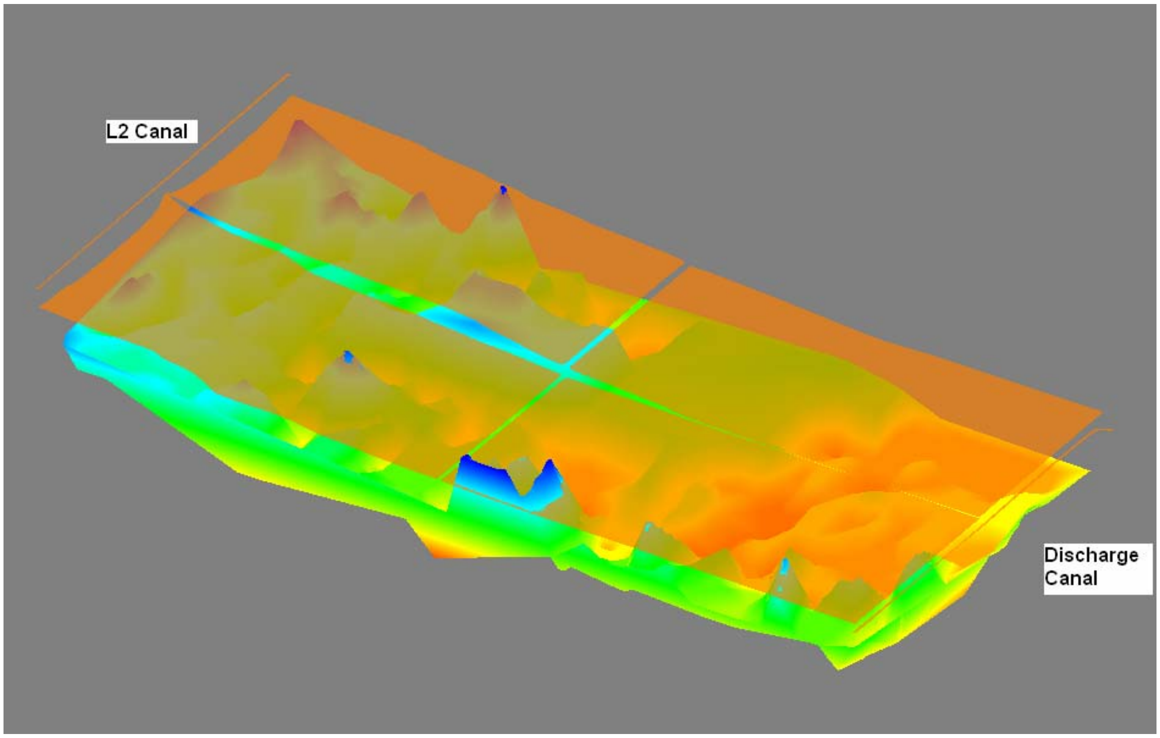

Figure 15: Water Surface Elevation Overlaying 3D Surveyed Land Surface Elevations (Design Flow) (Vertical scale not to scale)

3.3 Low Flow

- Total STA-5 inflow from L2 Borrow Canal is 200 cfs.
- No G-406 diversion.
- The seepage return flow is 50 cfs (2 x 25).
- ET: 0.24 inch/day
- Levee Seepage losses: 60 cfs.

Due to frequent dry-out of local high topographic points, numerical instabilities were encountered during the Low Flow simulation. Rainfall was considered negligible; ET was assumed to be 0.24 inch/day based on available observed ET data at STA-1W and levee seepage losses were assumed to be 60 cfs, 30% of the G-342A-D inflow. G-343A-H interior culverts and outflow structure G-344A-D were assumed to be fully open.

The downstream boundary condition was set as G-344A_T = 13.5 ft NGVD in the Discharge Canal. The inflow into the L2 borrow canal was 200 cfs.

A final simulation result was obtained, after using the results of a steady flow simulation (total inflow: 200 cfs and without explicitly incorporating culverts) as the initial condition, and running a transient simulation with all the 16 culverts in place. In addition, the frequent dry-out areas were set as inactive (disabled) elements in the final model run.

The water surface elevations (Figure 16) in Cell 2B range from 13.80 ft NGVD to 13.50 ft NGVD. In Cell 1B, water levels are close to a static value of 13.50 ft NGVD (ranging from 13.50 to 13.55). These two cells are designed as SAV dominant. As for the upstream treatment cells, Cell 1A and Cell 2A, there are significant water surface buildup at the spreader canals during the simulation. At least one quarter of Cell 1A treatment area is dry-out (ground elevations > 14.5 ft NGVD). Cell 2A has some local dry-out areas and water levels are lower than in Cell 1A, ranging from 13.60 ft NGVD to 15.0 ft NGVD. It appears that G-343A-H gate operations are needed to avoid dry-out in Cell 1A.

Further confirmed from the velocity magnitude plot (Fig. 18) and water depth distribution (Fig. 17), there are some ineffective treatment areas under the 200 cfs - Low Flow condition. These areas are the higher land surface elevation areas in the southern part of Cell 2B, the area upstream of G-343C and G-343D in Cell 1A and some local areas in Cell 2A. The dry-out areas are clearly shown in Figures 19.

If the specified stage in the Discharge Canal is lowered to 13.0 ft NGVD, then west portion of Cell 1B will be very close to dry-out and southern portion of Cell 2B will be totally dry-out. This indicates that when headwater levels at G-344A-D are below 13.50 ft NGVD, G-344 A-D may need to be closed to raise water levels in Cell 1B and Cell 2B.

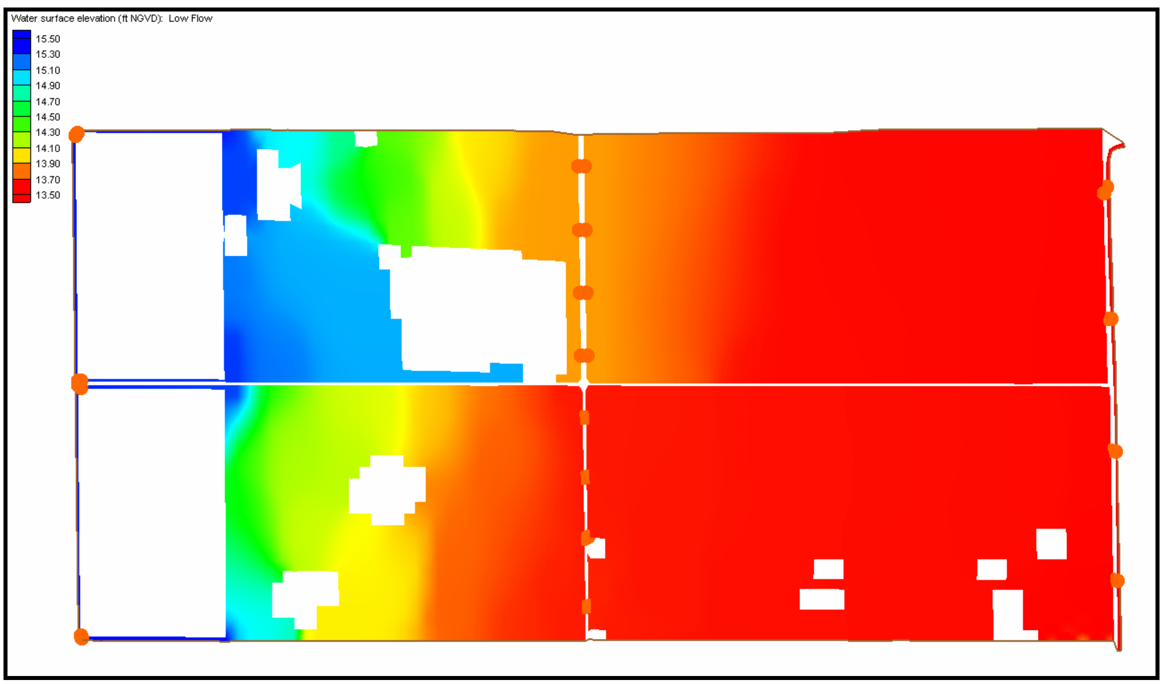

Figure 16: Water surface elevations (Low Flow)

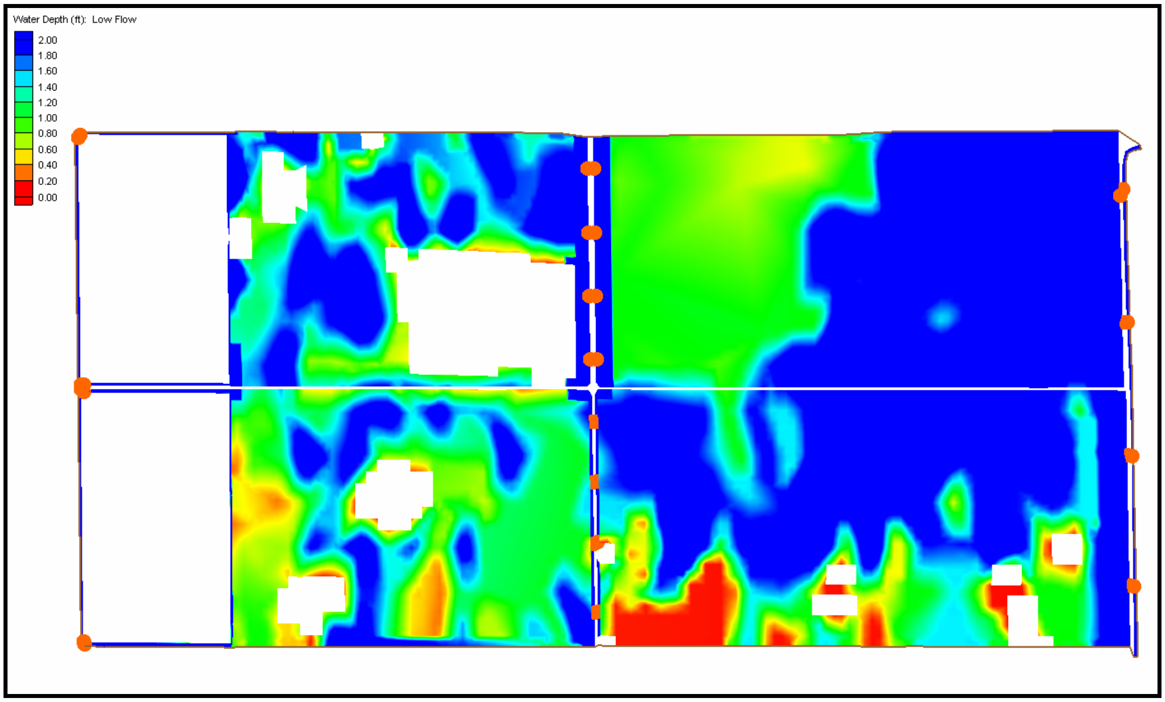

Figure 17: Water depth under Low Flow (200 cfs)

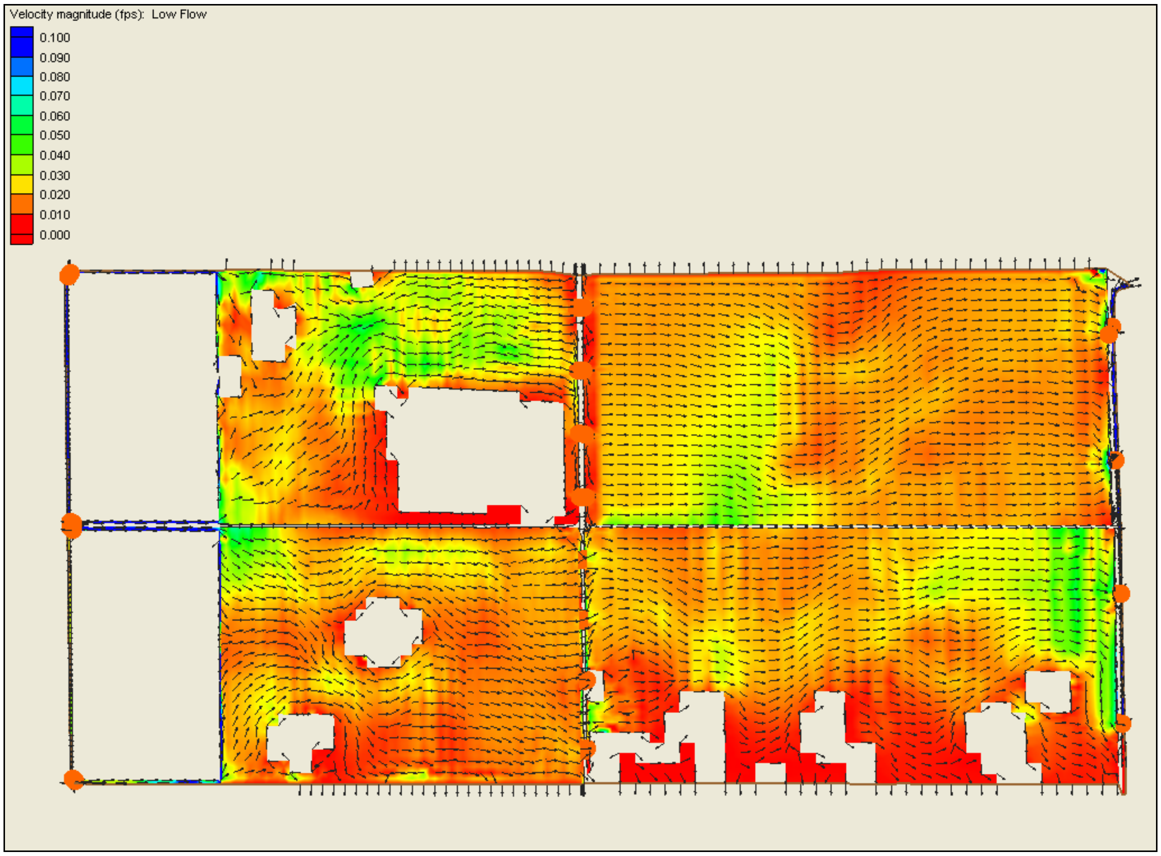

Figure 18: Velocity magnitude under Low Flow (200 cfs)

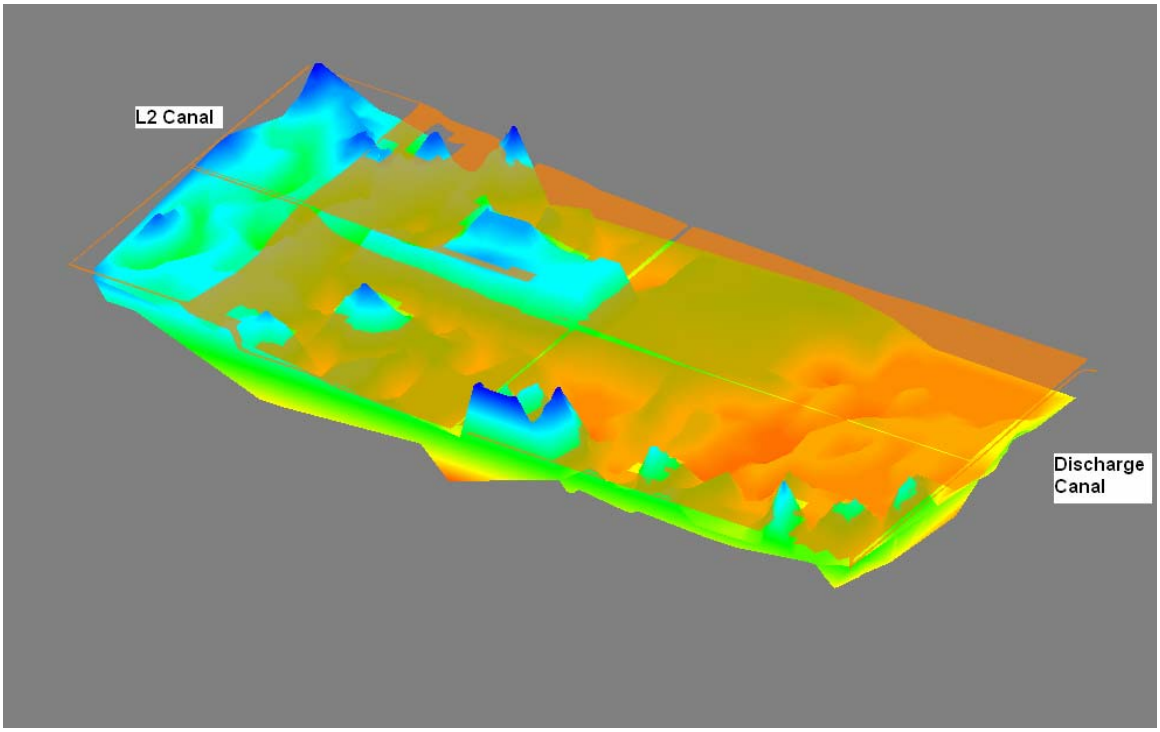

Figure 19: Water Surface Elevation Overlaying 3D Surveyed Land Surface Elevations (Low Flow) (Vertical scale not to scale)

3.4 High Flow

The High Flow condition was defined as (Standard Project Storm + structure inflow). Flood flow from the C-139 Basin into The L2 Borrow Canal is expected to be 3,440 cfs.

There are two cases for STA-5 Standard Project Storm flood based on the downstream Miami Canal flood condition.

- Case 1: Miami Canal in flood mode; G-344A TW is 13.6 ft NGVD.
- Case 2: Miami Canal not in flood mode; G-344A TW is 16.12 ft NGVD.

When the Miami Canal is in flood mode, it cannot receive excessive flood water from STA-5 and only 1,080 cfs of stormwater will be routed into STA-5. The remaining water volume (3,440- 1,080 =2,360 cfs) will be diverted by G-406 to the south.

The temporal distribution of the Standard Project Storm rainfall (21.6 inches in 24 hours) was obtained from (URS Corp, 2005), as shown in Figure 20.

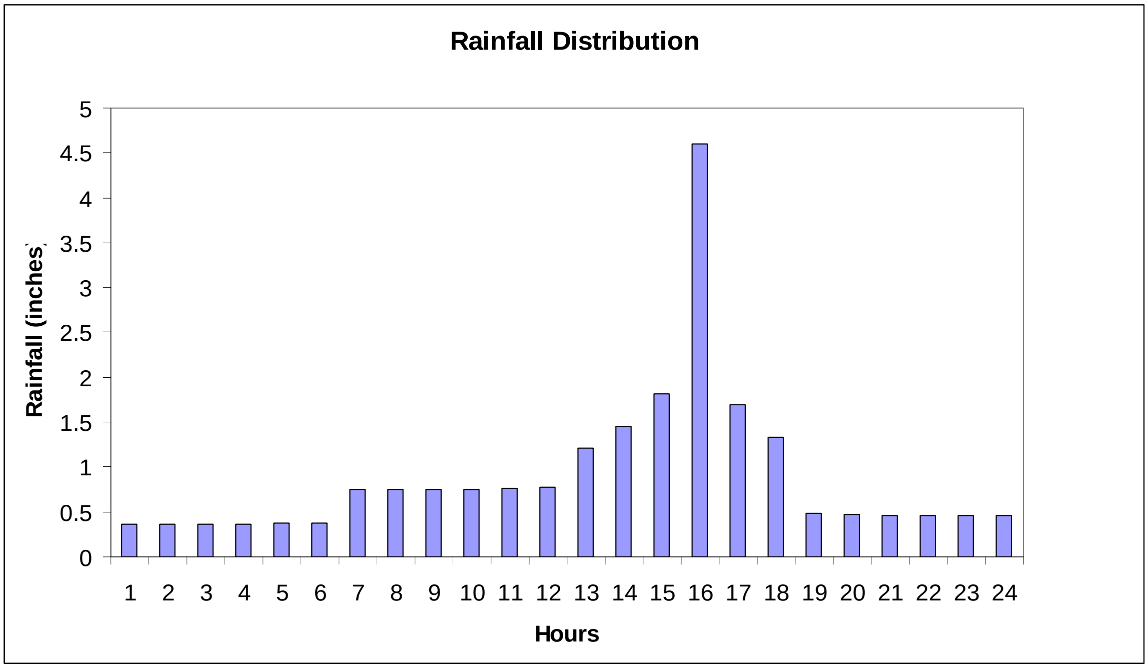

Figure 20: Rainfall distribution for SPS rainfall

FESWMS/Flo2DH does not have direct input for rainfall; as a result, point source terms were applied relatively uniform across the model domain in 200 nodes to mimic the direct SPS rainfall on the STA-5 surface area.

Transient simulations for High Flow conditions used the Design Flow condition as the initial condition. The downstream boundary conditions at G-344A-T in the Discharge Canal were obtained from the STA-5 Operation Plan as specified stages considered as the desired water levels for Miami Canal described as above.

In Figure 21, stage hydrographs during a SPS storm event is plotted for both the northern flow way (G-343C headwater and tailwater levels) and the southern flow way (G-343G).

The peak water levels are below 18.3 ft NGVD in Cell1A and Cell 2A, and below 17.7 ft NGVD in Cell 1B and Cell 2B if Miami Canal is not in flood mode and most of the SPS stormwater is routed into STA-5. These stages are within STA-5 design values.

The spatial distributions of water depth, velocity magnitude and water levels at the end of the SPS storm (Figures 22-24) show that water depth exceeds 4.5 ft about 50% of effective treatment areas; this may be harmful to the vegetations. Velocity magnitude values are mostly below the 0.1 ft/s threshold.

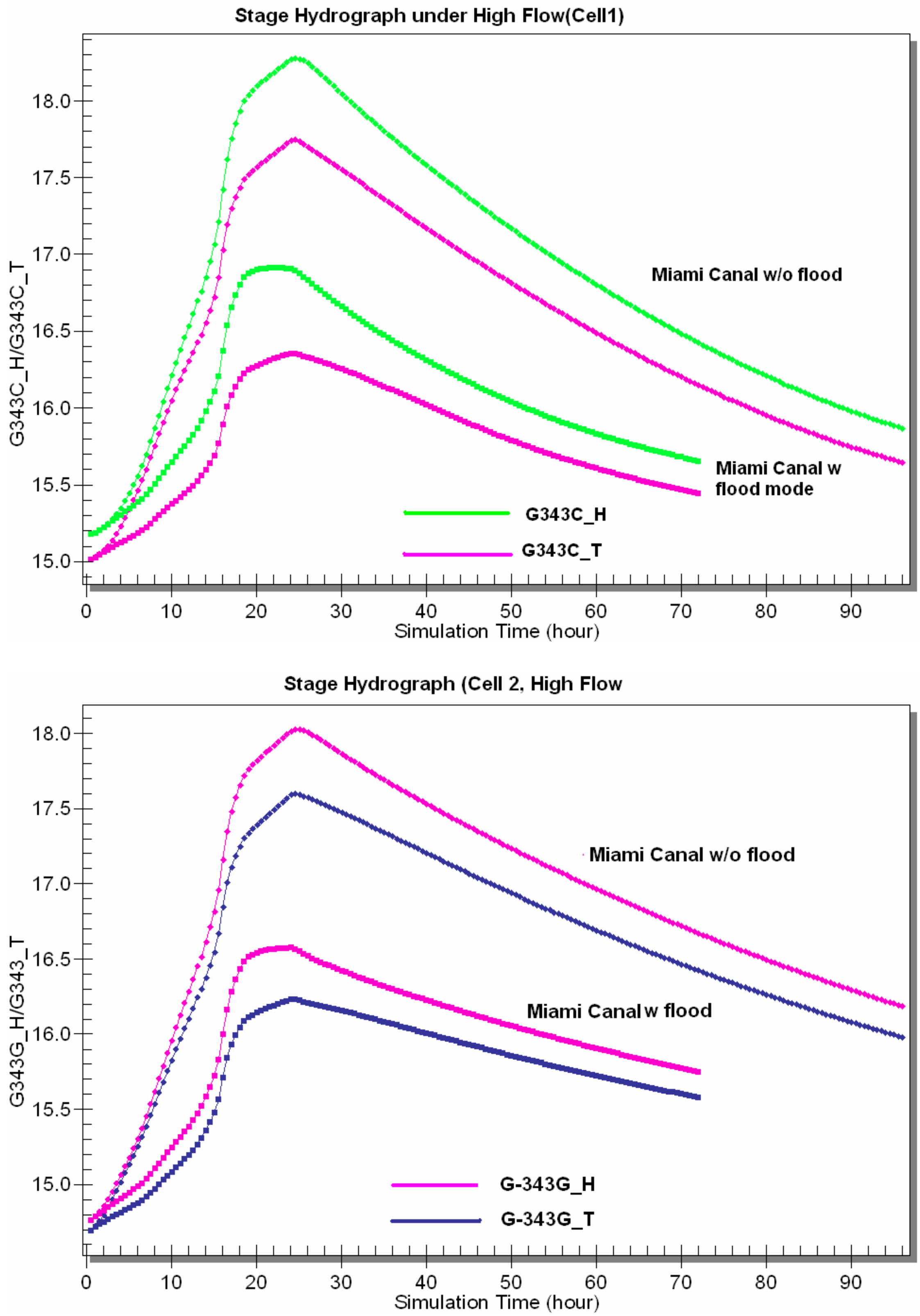

Figure 21: Stage Hydrograph under High Flow (G343 headwater and tailwater)

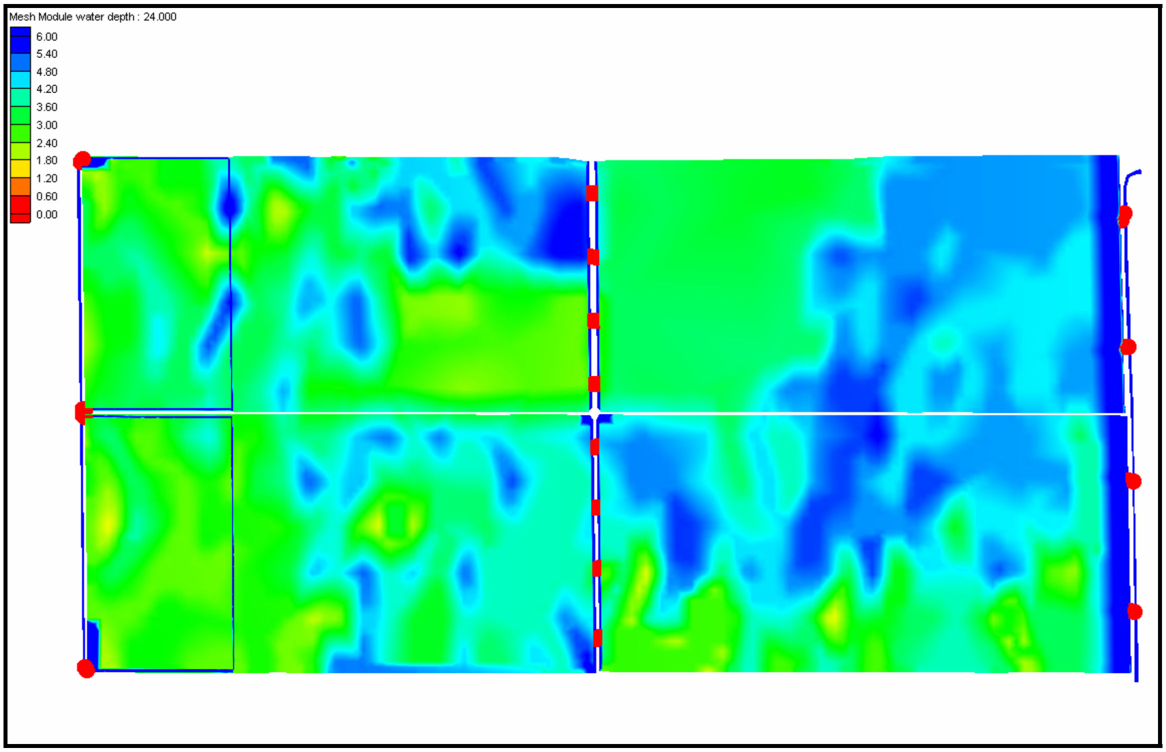

Figure 22: Water Depth Distribution at Peak Water Levels

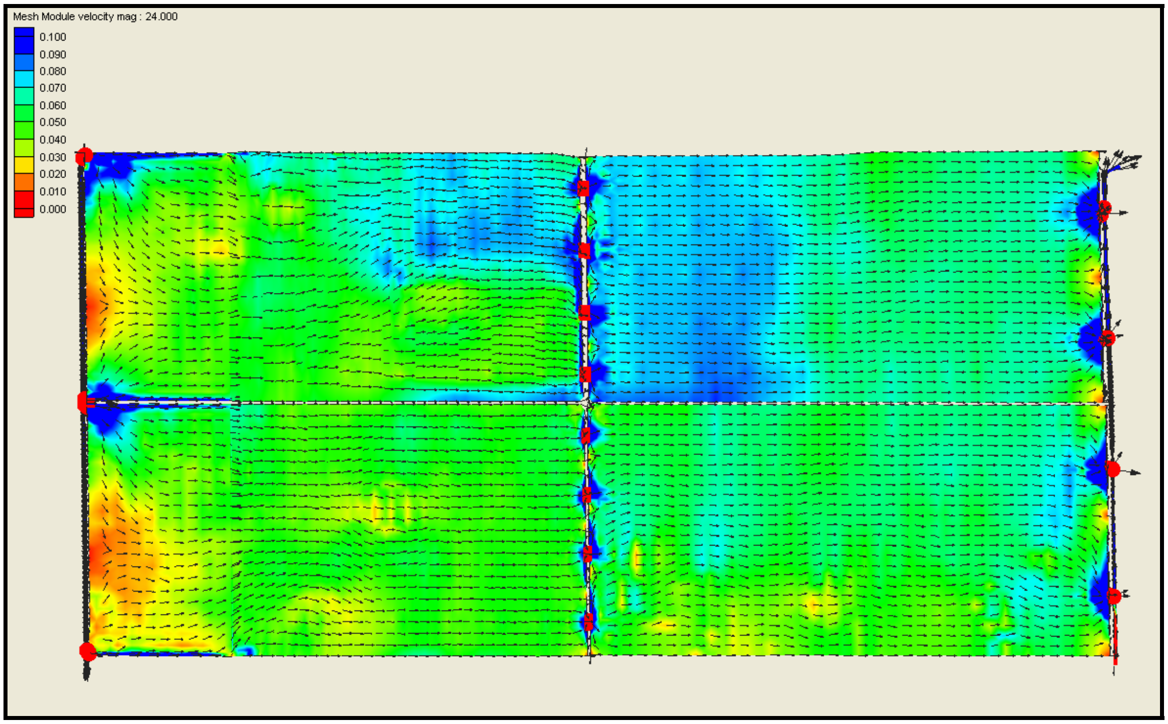

Figure 23: Velocity Magnitude Distribution at Peak Water Levels

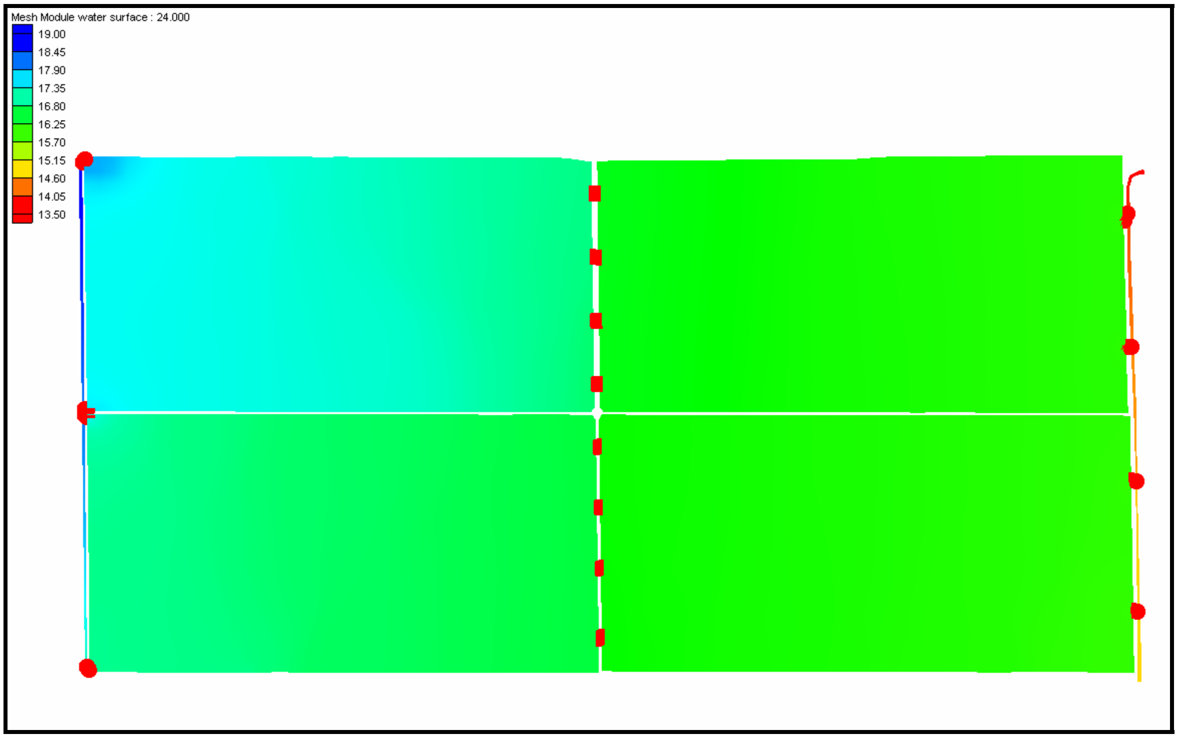

Figure 24: Peak Water Surface Elevations

3.5 Hydraulic Residence Time

The nominal hydraulic residence time (HRT) was determined by:

$$T = V/Q$$

T is hydraulic residence time (s);
V is total water volume in STA-5 under steady flow condition (ft^3);
Q is flow rate (G-342A-D+seepage return pumps) (cfs).

The nominal hydraulic residence time under Design Flow Condition is 5.6 days. The nominal hydraulic residence time for Low Flow Condition is 16.8 days.

A few more steady flow simulations with flow rates between 250 cfs and 1,250 cfs were made and the relationship between flow rate and hydraulic residence time was obtained and plotted in the following Figure 25. The assumptions made in these steady state flow simulations are that seepage return pumping rate (G-349A and G-350A) was maintained at 25 cfs for each structure; levee seepage rate was 20% of G-342A-D inflow and water level in the Discharge Canal is maintained at 14.13 ft NGVD (G-344A tailwater level).

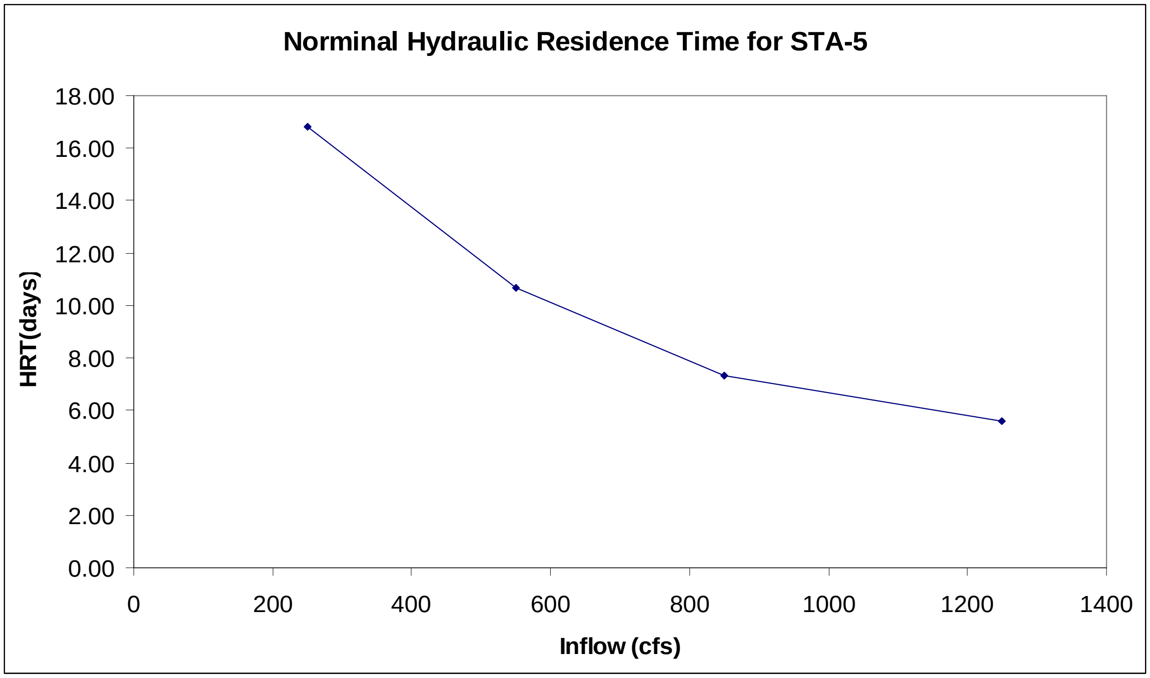

Figure 25: Relationship between Flow Rate and Hydraulic Residence Time.

4. Sensitivity Analysis

A sensitivity analysis was completed to investigate the range of system output sensitivity due to variation in model parameters (e.g., Manning's roughness coefficient and seepage losses).

The Base Case is Design Flow condition simulation, with the Manning's n values in Table 2 and levee seepage losses were considered as 20% of structure inflow applied along the levees (-240 cfs).

The following four scenarios were simulated and analyzed:

- Case 1: Manning's n values in the marsh area being increased by +30%
- Case 2: Manning's n values in the marsh area being decreased by -30%
- Case 3: No levee seepage applied.
- Case 4: Constant Manning's n values applied in the marsh area (cattail: 0.5, SAV: 0.3).

In comparing simulation results, the two transects defined in Figure 10 (A-A' and B-B') will be used.

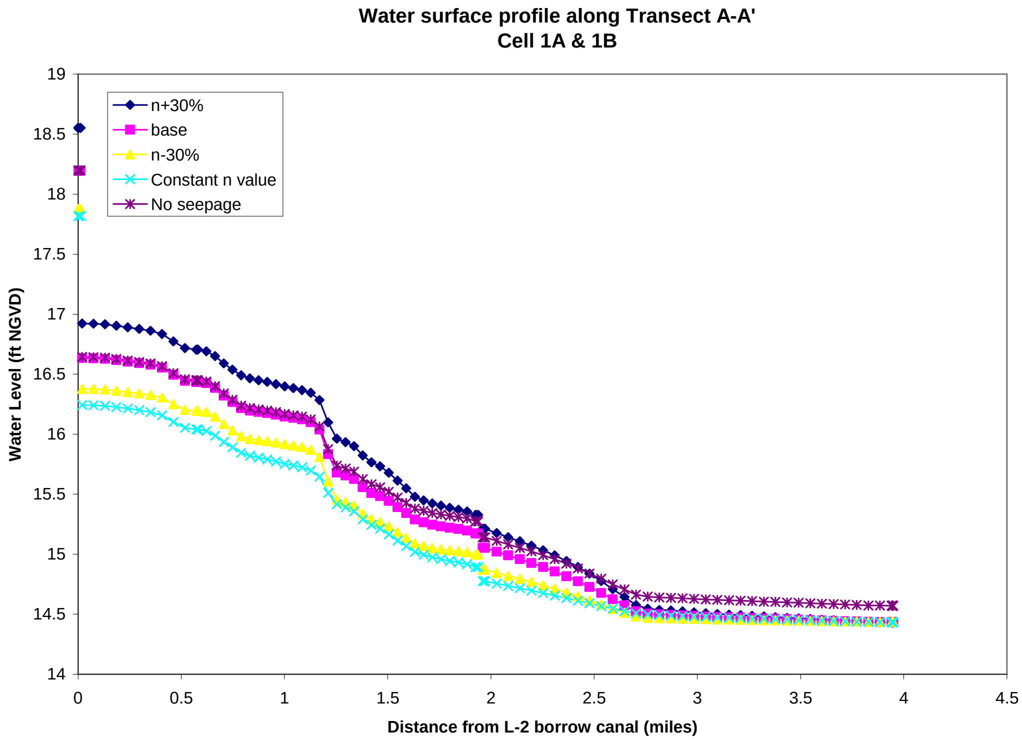

Figure 26: Water surface profiles for different cases (A-A' in Cell 1)

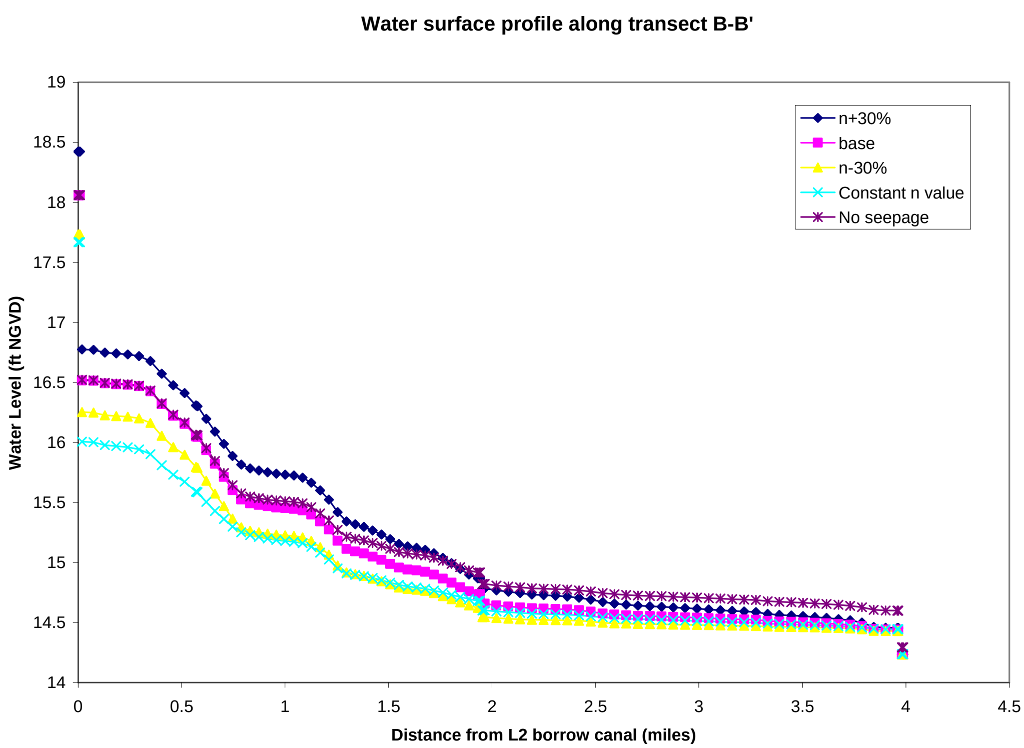

Figure 27: Water surface profiles for different cases (B-B' in Cell 2)

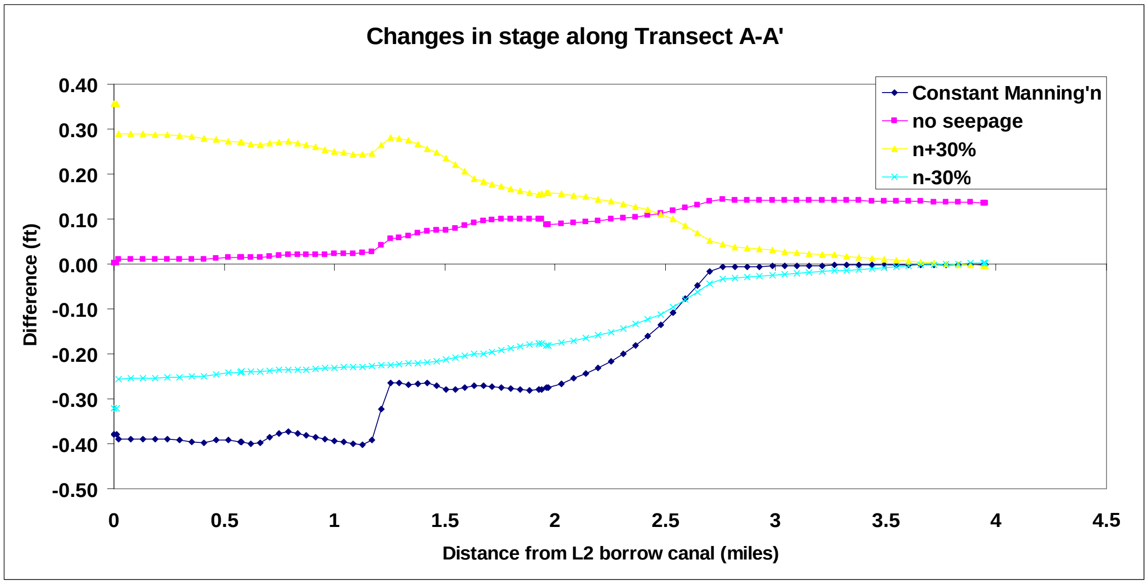

Figure 28: Changes in water levels along A-A' (Cell 1)

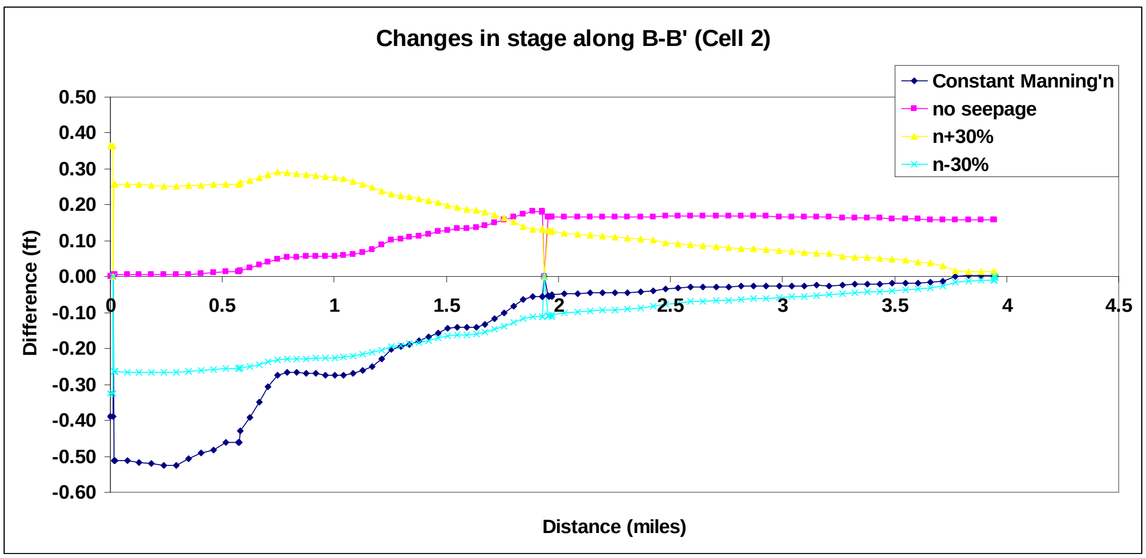

Figure 29: Difference in water level along B-B' (Cell 2)

Results of Sensitivity Tests

±30% of variation in Manning's roughness coefficient (cattail and SAV) caused a maximum difference of ±0.3 ft in water depth along A-A' and B-B' (Cell 1 and Cell 2), this is indicated in Figures 26-29. When constant Manning's n values are used instead of a depth-dependent relationship, water levels dropped markedly in the west portion of Cell 1A and Cell 2A. This is consistent with the small water depth in this area.

The velocity magnitude distribution in the marsh area is not significantly different when Manning's n values were changed by ±30% (Figure 30). The maximum change in velocity magnitude is for a local area at the southeastern corner of Cell 1A, upstream of G-343C and G-343D.

Inclusion of levee seepage losses draws down Cells 1B and 2B water levels by about 0.2 ft. In the Transect plots, the south-north interior levee is located at about 2 miles from L2 borrow canal.

From these sensitivity simulation results, we believe that the pre-selected Manning's n values are reasonable. Even though the true value of Manning' n values for STA-5 vegetation could have a ±30% deviation from the used values, the conclusions on flow distribution, flow velocity pattern and water surface elevations will not be significantly different.

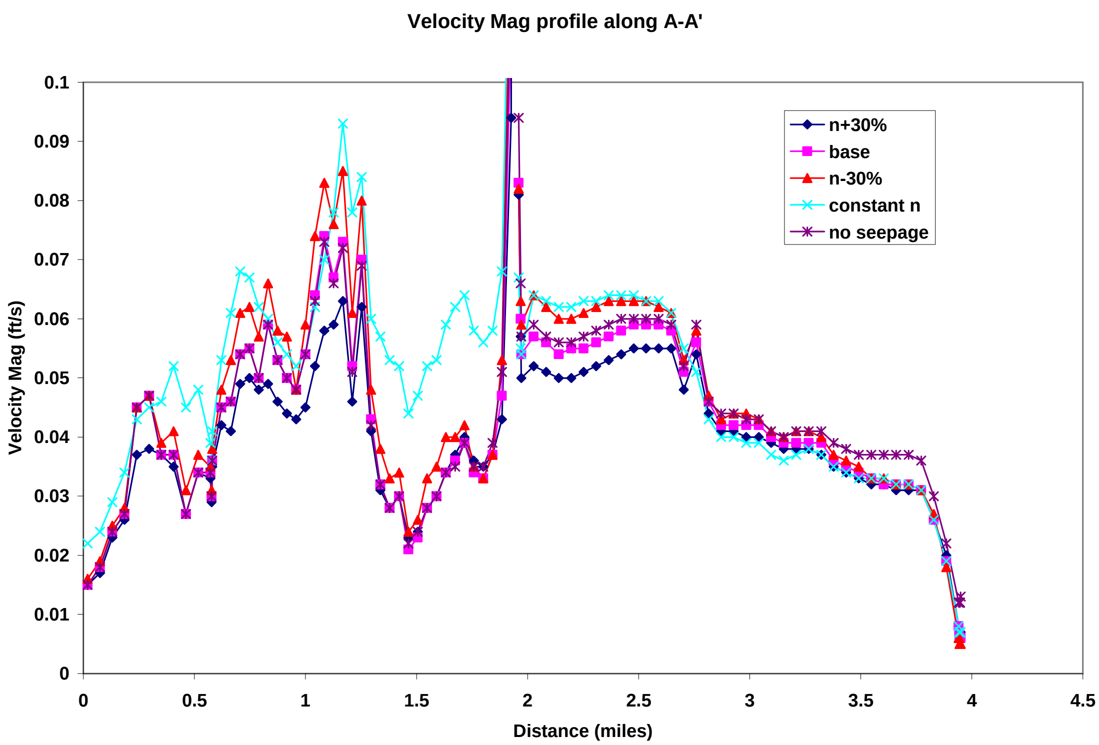

Figure 30: Comparison of velocity magnitude

5. Discussions and Conclusions

Higher land surface elevations in some local areas may impact treatment efficiency. For example, high topographic areas upstream of G-343C and D have an average land elevation of 14.5 ft NGVD. Model simulations show that the water depth in these locations is about 0.3 to 1.0 ft in the Design Flow condition. It appears that these areas are very shallow or even dry under normal operation. In Figure 31, areas with ground surface elevation greater than 12.50 ft NGVD are shown. The southern portion of Cell 2B has some local high land surface elevation regions and may be dry under normal operation. This is undesired when SAV is the dominant vegetation in Cell 2B.

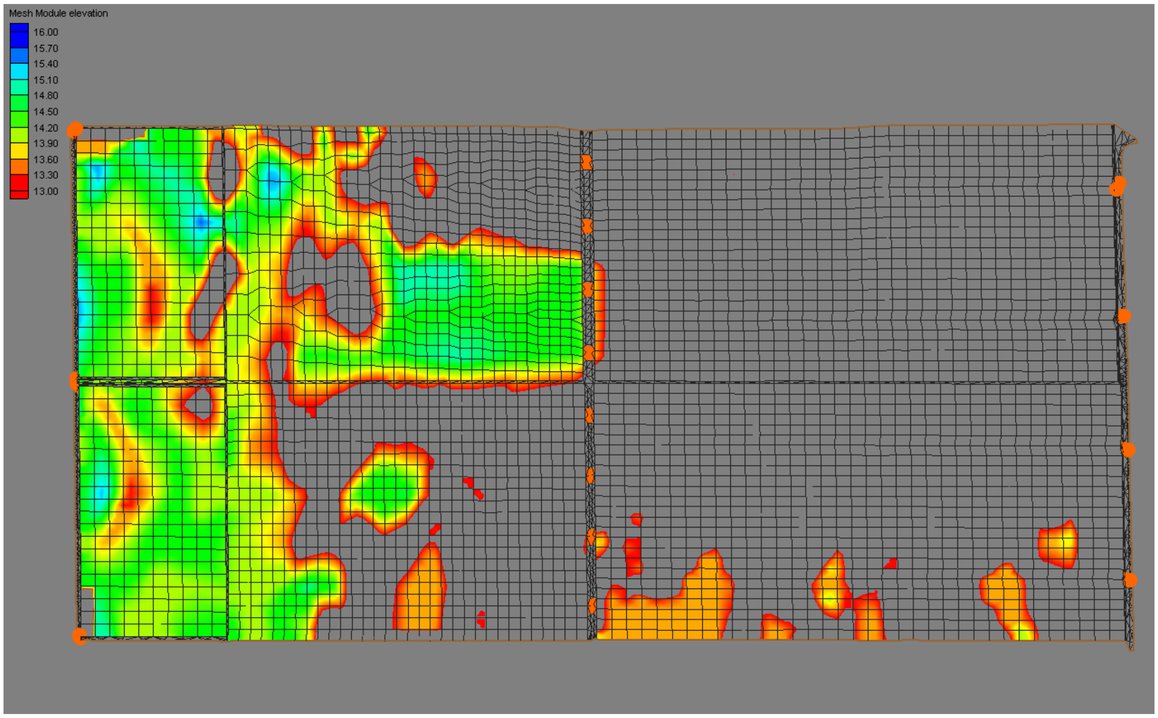

Figure 31: STA-5 ground surface elevations (those areas higher than 13.0 ft NGVD)

In summary, a new two-dimensional linked cells hydraulic model has been built for STA-5. The L2 borrow canal, four treatment cells and the Discharge Canal were simulated and linked together by sixteen culverts.

Since the developed STA-5 linked cells model represents the future enhanced configuration of STA-5, model parameters, mainly Manning's roughness coefficient, were not able to be calibrated with historical flow and stage data. It is recommended that model calibration will be performed when STA-5 enhancements are completed and new observed flow and stage data are available.

Low Flow, Design Flow and High Flow scenarios were simulated with the new linked cells model. Although there are no obvious short-circuiting flow ways, there are some local high surface elevation areas that may experience frequent dry-outs and compromise STA-5 treatment efficiency.

Sensitivity test results demonstrate that changing Manning's n values for the marsh areas by 30% caused less than 5 inches of changes in computed water levels; the velocity magnitude distribution had small changes except those areas with small water depth. The depth-dependency assumption for Manning' n values has obvious impact on flow distribution.

By running a series of steady flow simulations, a relationship between nominal hydraulic residence time and inflow rate was obtained. The nominal hydraulic residence time ranges from about 6 days to 17 days, depending on STA-5 inflow.

We believe that the new two-dimensional linked cells hydrodynamic model is a significant improvement from previous STA-5 single cell steady flow models. All culverts are explicitly represented in the new model. If the limitation in gate representation can be overcome, this will be a good modeling tool for STA-5 operation and management.

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